MERRIMACK RIVER BASIN

BELMONT, NEW HAMPSHIRE

LOCHMERE DAM

NH 00015

NHWRB 21.07

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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NEW ENGLAND

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam is 223 ft. long and about 14 ft. high. The dam is intermediate in size with a hazard potential of significant. The dam is on good condition at the present time. Only a few relatively minor operating and maintenance improvements are necessary. The owner should implement a formal, written flood and emergency warning system.

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:

NEDED

JAN 23 1979

Honorable Hugh J. Gallen
Governor of the State of New Hampshire
State House
Concord, New Hampshire 03301

Dear Governor Gallen:

I am forwarding to you a copy of the Lochmere Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Water Resources Board, the cooperating agency for the State of New Hampshire. In addition, a copy of the report has also been furnished the owner, New Hampshire Resources Water Board, 37 Pleasant Street, Concord, New Hampshire 03301, ATTN: Mr. George M. McGee, Sr., Chairman.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Water Resources Board for your cooperation in carrying out this program.

Sincerely yours,

John P. Chandler

JOHN P. CHANDLER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

LOCHMERE DAM
NH 00015

MERRIMACK RIVER BASIN
BELMONT, NEW HAMPSHIRE

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

Identification No.:	NH 00015
NHWRB No.:	21.07
Name of Dam:	LOCHMERE DAM
Town:	Belmont
County and State:	Belknap, New Hampshire
Stream:	Winnipesaukee River
Date of Inspection:	May 31, 1978

BRIEF ASSESSMENT

Lochmere Dam is a 223 foot long, concrete and stone gravity dam with a maximum height of approximately 14 feet. The dam consists of, beginning with the left bank, a 13 foot long concrete abutment, a 72 foot long, six bay spillway and sluice gate structure, a 72 foot long, six bay spillway with provision for stoplogs, a 17 foot long, three bay ogee spillway, a 17 foot long intermediate pier supporting a gatehouse and a 38 foot long, five bay sluice gate structure. The dam, which is owned by the New Hampshire Water Resources Board (NHWRB), appears to be founded on dense glacial till. The original dam was built in 1910, modified in 1957 and further altered to its present configuration in 1976.

The dam, which lies on the Winnipesaukee River and impounds Lake Winnisquam, is used primarily to maintain the lake for recreational purposes, with a secondary function as a flood control structure. The 428 square mile drainage area of gently to steeply sloping forest includes the 363 square mile Lake Winnipesaukee drainage area and the 11 square mile Opechee Bay drainage area. The dam's maximum impoundment of 33,280 acre-feet places it in the INTERMEDIATE size category, while the possibility of heavy property damage, but unlikely loss of life, in the event of failure indicates a SIGNIFICANT hazard potential classification.

Based on the size and hazard potential ratings and in accordance with the Corp's guidelines, the Test Flood (TF) is one-half the Probable Maximum Flood (PMF). Because this dam is part of a complex system of dams, lakes and channels which comprise the Winnipesaukee River drainage basin, the PMF cannot be readily determined. Using an analysis within the scope of a Phase I investigation, however, a TF inflow of 27,000 cfs yields an inflow at the dam of 10,000 cfs.

The dam's maximum discharge capacity is only 7800 cfs, or 78% of the Test Flood and, thus, the dam could be overtopped by as much as 2 feet. Based on this analysis, an improvement in the dam's discharge capacity is recommended.

The dam is in GOOD condition at the present time. Only a few relatively minor operating and maintenance improvements are necessary. Included in these are modification or replacement of the present hand crank system so that gates can be operated manually, monitoring of erosion at the end of the right training wall and of seepage through the square stone masonry near the left sluice gates when the gates are open, replacement of inadequate stoplogs, installation of a gauge at the dam and training of local officials in the dam operations to decrease response time in the event of emergencies. Additionally, the owner should implement a formal, written flood and emergency warning system.

The above recommendations and remedial measures should be implemented within 2 years of receipt of the Phase I Inspection Report by the owner. In light of the dam's GOOD condition, periodic technical inspections should be accomplished every two years.

THE STATE OF NEW HAMPSHIRE
WILLIAM S. ZOINO
No. 3226
REGISTERED PROFESSIONAL ENGINEER

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This Phase I Inspection Report on Lochmere Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Richard F. Doherty

RICHARD F. DOHERTY, MEMBER
Water Control Branch
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

Joseph A. McElroy

JOSEPH A. MCELROY, CHAIRMAN
Chief, NED Materials Testing Lab.
Foundations & Materials Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the Test Flood should not be interpreted as necessarily posing a highly inadequate condition. The Test Flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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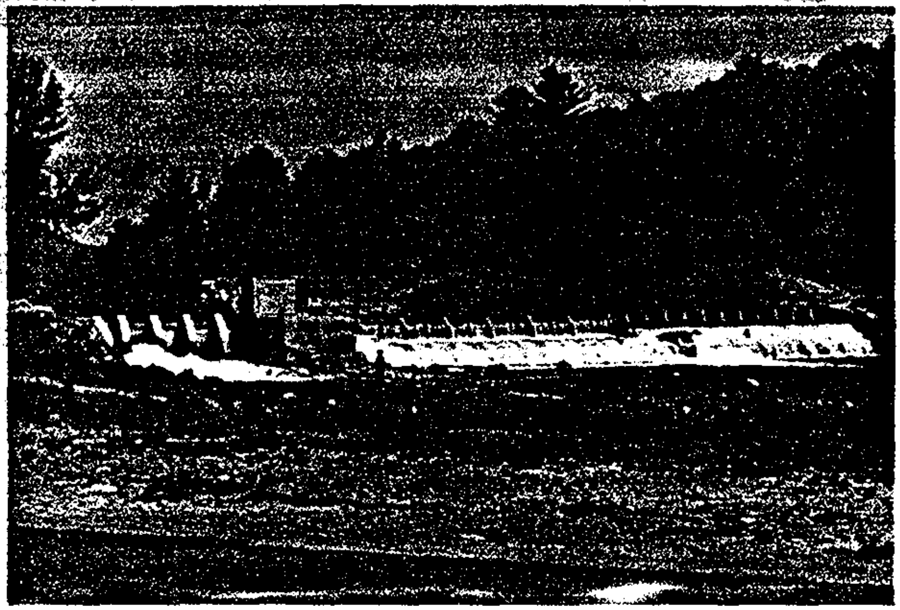
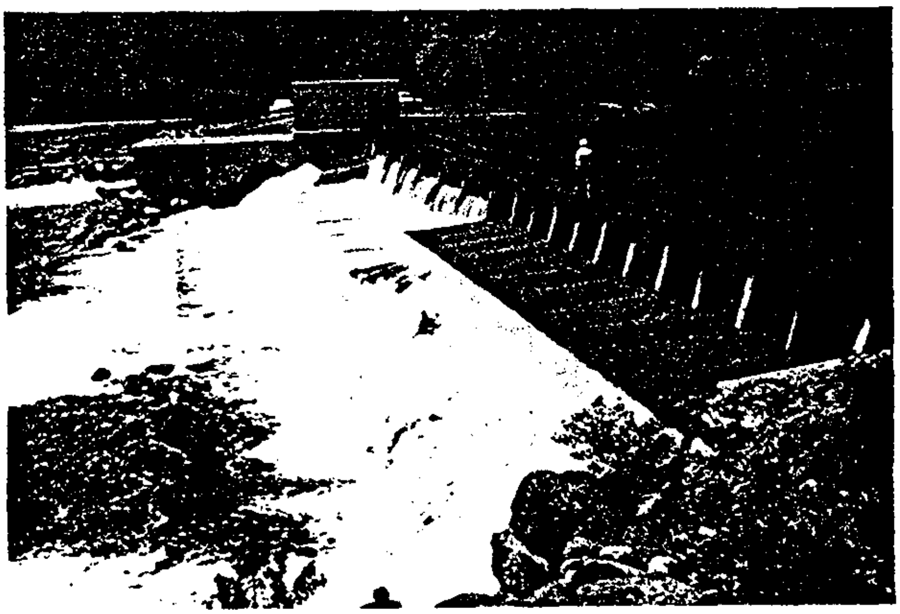


Overview from right side downstream



Overview from left abutment

FROM: USGS PENACOCK, N.H.
QUADRANGLE MAP

GOLDBERG, ZOINO, DUNNICLIFF & ASSOC, INC. GEOTECHNICAL CONSULTANTS NEWTON UPPER FALLS, MASS.	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCUS PLAN

LOCHMERE DAM — NEW HAMPSHIRE

SCALE AS NOTED
DATE SEPT 1978

FILE No. 2067

PHASE I INSPECTION REPORT

LOCHMERE DAM

SECTION I

PROJECT INFORMATION

1.1 General

(a) Authority

Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of dam inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZD) has been retained by the New England Division to inspect and report on selected dams in the State of New Hampshire. Authorization and notice to proceed was issued to GZD under a letter of August 22, 1978 from Colonel Ralph T. Garver, Corps of Engineers. Contract No. DACW 33-78-C-0303 has been assigned by the Corps of Engineers for this work.

(b) Purpose

(1) Perform technical inspection and evaluation of non-federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-federal interests.

(2) Encourage and prepare the states to initiate quickly effective dam safety programs for non-federal dams.

(3) Update, verify and complete the National Inventory of Dams.

(c) Scope

The program provides for the inspection of non-Federal dams in the high hazard potential category based upon location of the dams and those dams in the significant hazard potential category believed to represent an immediate danger based on condition of the dams.

1.2 Description of Project

(a) Location

The Lochmere Dam lies on the Winnipesaukee River approximately 0.4 miles southeast of the village of East Tilton, N.H. The site is easily accessible via Routes 3/11. The portion of the USGS Penacook, NH quadrangle presented previously shows this locus. Figure 1 of Appendix B presents a detail of the site developed from the inspection visit and the map.

(b) Description of Dam and Appurtenances

This dam, approximately 223 feet in length, consists of six basic structural components (Fig. 2). Beginning at the left bank, the components consist of a concrete abutment 13 feet long, a combination spillway and sluice gate structure 72 feet long, a 6 bay spillway 72 feet long equipped with stop logs, a 3 bay spillway approximately 17 feet long, an intermediate pier approximately 11 feet long which supports a service building, a five bay sluice gate structure 38 feet long (Fig. 3) and a combination wingwall and training wall approximately 140 feet long (forming the right abutment). Steel grating service bridges span over the water control structures on either side of the service building. The dam's maximum height is approximately 14 feet.

The right side of the dam formerly discharged into a channel which carried the water to a hydroelectric plant approximately 400 feet downstream. The plant, however, is no longer in service and the channel is now backfilled. (Fig. 4)

Borings executed in March 1976 in preparation for construction of the 5 bay sluice gate structure indicate that the dam may be founded on at least 10 feet of very dense, bouldery glacial till.

(c) Size Classification

The dam's maximum impoundment of 33,280 acre-feet falls within the 1000 acre-feet to 50,000 acre-feet range which defines the INTERMEDIATE size category as outlined in the "Recommended Guidelines."

(d) Hazard Potential Classification

Flow through Lochmere Dam travels in a wide, shallow channel for approximately 500 feet before entering Silver Lake. The shallow channel and lake would serve to dampen any flood wave generated at the dam. For this reason, the potential for loss of life in the event of a failure is probably low. Rising waters would, however, cause considerable property damage to low lying structures around Silver Lake and along the course of the river to Tilton. Therefore, a hazard potential classification of SIGNIFICANT is appropriate for this dam.

(e) Ownership

The New Hampshire Water Resources Board (NHWRB) owns this dam. Key officials of the Board are; Chairman George McGee, Chief Engineer Vernon Knowlton, Assistant Chief Engineer Donald Rapoza and Staff Engineer Gary Kerr. The Board has offices at 37 Pleasant Street, Concord, N.H. 03301, and can be reached by telephone at (603) 271-3406 or (603) 271-1110. The Public Service Company of New Hampshire turned the structure over to the state in 1966.

(f) Operator

The NHWRB has a permanent dam tender who operates the Lakeport, Avery and Lochmere Dams and several smaller structures. He receives instructions daily from the Board's offices in Concord and can be contacted through the Board.

(g) Purpose of Dam

The primary purposes of the dam are to regulate the level of Winnisquam Lake for recreational purposes and to provide some flood protection along the Winnipesaukee River.

(h) Design and Construction History

Historical records indicate that initial construction occurred in 1910. The original structure, built by the Public Service Company of New Hampshire (PSCNH), retained water for power generation at the abandoned hydroelectric plant 400 feet downstream.

In 1957, PSCNH made some alterations to the 161 foot long section of the dam between the intermediate pier and the left abutment. Significant alterations took place in 1976 when the NHWRB constructed the 5 bay sluice gate structure on the right side of the dam and backfilled the channel to the hydroelectric plant which had been idle for at least 10 years (Fig. 4).

(i) Normal Operational Procedures

The NHWRB operator visits the dam at least every other day and reports gage readings back to the Concord office. Engineers at the head office, in turn, direct any gate operations necessitated by the operator's input. In late summer, the Board draws the lake down 2 feet in anticipation of fall storms and spring runoff.

1.3 Pertinent Data

(a) Drainage Areas

The Lochmere Dam must pass flow from the Lake Winnepesaukee drainage area, some 363 square miles, plus the Opechee Bay and Lake Winnisquam drainage areas of 65 square miles. The upstream Avery and Lakeport dams, however, also assist in the control of the Lake Winnipesaukee discharges. In general, the terrain is forested and gently sloping, although regions of steep terrain border the lakes at some points. The area is a major recreational center and, as such, has considerable development all around both lakes and on the many islands in Lake Winnipesaukee.

(b) Discharge at Dam Site

(1) Outlet Works

The outlet works at the dam consist of the six 4 feet, 1 inch wide by 2 feet 9 inch high gated tunnels and the five 6 feet wide by 6 feet high sluice gates. Both sets of features have inverts at El. 471.3.

(2) Maximum Known Flood at Damsite

Records for the USGS streamflow gauge at Tilton, New Hampshire (Gauge No. 01081000) extend back at least 40 years.

The peak flow at the gauge occurred during the September 1938 hurricane when a flow of 3810 cfs was recorded. The gauge was not in operation during the March 1936 flood.

(3) Spillway capacity at maximum pool elevation (includes 3 bay spillway and two 6 bay spillways): 2400 cfs at El. 484

(4) Gate capacity at normal pool elevation (includes 5 new sluice gates and 6 tunnels): 3500 cfs at El. 482

(5) Gate capacity at maximum pool elevation: 3800 cfs at El. 484

(6) Total discharge capacity at maximum pool elevation: 6200 cfs at El. 484

(c) <u>Elevation</u> (ft. above MSL)

(1) Top of dam (walkway): 484.4

(2) Maximum pool: 484 ±

(3) Recreational pool: 482 ±

(4) Spillway crest: 481.3

(5) Streambed at centerline of dam: 471 ±

(6) Maximum tailwater: Unknown

(d) <u>Reservoir</u>

(1) Length of recreational pool: 10 miles ±

(2) Storage of recreational pool: 20,800 acre-feet ±

(3) Storage of maximum pool: 33,280 acre-feet ±

(4) Area of reservoir: 4160 acres ±

(e) Dam

(1) Type: Concrete and stone gravity

(2) Length: 223 feet

(3) Height: 14 feet structural
13 feet hydraulic

(4) Top width: Varies to as much as 23 feet

(5) Side slopes: Upstream - vertical
Downstream - varies to as flat as 3:1

(6) Cutoff and grout curtain: Unknown

(f) Spillway

(1) Type: Concrete broad crested

(2) Length of weir: 161 feet

(3) Crest elevation: 89 linear feet at El. 481.3
72 linear feet at El. 477.9

(4) Gates: 72 linear feet permit installation of up to 3 feet of stoplogs

(5) U/S channel: Open pond

(6) D/S channel: Concrete and stone aprons discharging into wide channel

(g) Regulating Outlets

Information concerning the number and size of regulating outlets is contained in subparagraph 1.3(b)(1) above. Rising stem mechanisms permit the operation of all 11 gates, either electrically or manually. The electrical operator is a portable device similar to an electric drill which receives power from a receptacle at the service building.

SECTION 2 - ENGINEERING DATA

2.1 Design Records

The design of the Lochmere Dam is straight forward and incorporates no unusual features. None of the original hydrologic, hydraulic or structural calculations are available, however.

2.2 Construction Records

None of the original construction plans, particularly those concerning foundation conditions or the precise nature of the old gravity structure which has since been altered, are available. On the other hand, existing plans for the 1957 and 1976 renovations adequately present most important features of the changes to the original dam.

2.3 Operational Records

The NHWRB operates the dam in a manner consistent with its intended purpose and engineering features and maintains satisfactory records of the dam's operation.

2.4 Evaluation

(a) Availability

Neither design calculations nor as-built drawings are available, if indeed they exist. While construction drawings concerning the alterations are available and generally detailed, the lack of design data and information on foundation conditions results in a marginal evaluation for availability.

(b) Adequacy

The lack of in-depth engineering data does not permit a definitive review. Therefore, the adequacy of this dam cannot be assessed from the standpoint of reviewing design and construction data. The assessment is thus based primarily on the visual inspection, past performance history and sound engineering judgement.

(c) Validity

Since the observations of the inspection team generally confirm the available written and verbal data, these sources of information warrant a satisfactory evaluation for validity.

SECTION 3 - VISUAL INSPECTION

3.1 Findings

(a) General

The Lochmere Dam is in GOOD condition at the present time and requires no immediate remedial measures for continued safe operation.

(b) Additional Description

Due to the large number of features incorporated into the dam, the following paragraphs expand on the description provided in subparagraph 1.2(b).

The dam between the left abutment and the intermediate pier consists of dry squared stone masonry. A concrete facing was placed on the upstream side sometime in the past. The top of the stone masonry dam is also capped with concrete and accomodates spillway crests. The downstream side of this structure is stepped, dry squared stone masonry. The upstream side of the original stone was near vertical faced. The length of this stone structure between the left abutment and the intermediate pier is approximately 161 feet.

The foundation for the intermediate pier, which varies from 11 feet to 17 feet wide and which is 50 feet in length, consists of concrete faced stone masonry with retained fill.

The left abutment is a concrete structure penetrating 13 feet into the left bank. A combination concrete and dry stone masonry rubble wingwall extends approximately 27 feet downstream from the abutment.

The combination sluice gate and spillway structure consists of 6 double spillway bays, 12 feet in width, with sluice gates centered in each bay and outlet tunnels offset at each bay. The spillway surface is broad crested. Each bay is subdivided by a concrete pier for supporting the service bridge. Rising type gate stems and crank operated, bench stand gear boxes are supported on structural steel yokes (twin channel sections) spanning over each bay; these yokes, in turn, are supported by means of steel columns 12 feet apart which frame into the concrete piers. The actual size of the sluice gates cannot be measured as the gates and guides are submerged. Historical data indicate that the gates are 5 feet high by 4 feet wide. Sluice gate outlets through the dam are also submerged due to normal tail-water conditions, but existing plans describe the outlets as "tunnels" 4 feet, 1 inch wide and 2 feet, 9 inches high.

The spillway portion of the dam with stoplogs consists of 6 double bay sections, 12 feet long, which are subdivided by structural steel supports for supporting the services bridge and steel stoplog guides. Stoplogs were in place to a height of 15 inches at the time of the inspection.

The spillway portion of the structure consists of an ogee section divided into 3 bays by means of two intermediate concrete piers. These piers support the service bridge.

The service building, which is located on the intermediate pier, is constructed of concrete block masonry units and reinforced concrete roof and floor slabs. The structure is 8 feet, 8 inches wide and 20 feet long.

The 5 bay sluice gate structure consists of 6 feet wide by 6 feet high timber gates with gear boxes and rising stems. Gear boxes are the crank operated, floor stand type and are supported on structural steel channel sections. Intermediate buttress type piers, 2 feet thick, support the steel channel sections and the service bridge. The right abutment, intermediate sluiceway piers and the right side of the pier supporting the service building have integrally cast stoplog slots both upstream and downstream of the sluice gates for maintenance purposes. This structure is also equipped with a full width, concrete energy dissipator approximately 29 feet downstream of the sluice gate axis. A full width concrete apron extends approximately 32 feet downstream from the energy dissipator and coincides with the downstream limits of the right training wall. There is evidence of a former training wall foundation extending downstream from the concrete faced intermediate pier.

(c) Dam

(1) Left Abutment

The concrete in this abutment and the wingwall extension is in good condition with no signs of spalls, cracking or efflourescence. The dry stone masonry rubble wingwall shows no evidence of bulging or displacement. No seepage around or under the location where the 13 foot long concrete wall penetrates the left bank was noted.

(2) Sluice Gate and Spillway Structure (Photo 2)

The squared stone masonry including the exposed stepped apron appears to be in good condition without any signs of displacement. Due to tailwater conditions and the flow over the spillway, physical observation of the "tunnel" outlets was impossible. However, when a representative of the NHWRB opened a sluice gate, some seepage appeared through the stone joints; the degree of seepage decreased as the discharge increased. Conversely, during the closing of this particular gate, the amount of seepage increased as the discharge decreased.

The concrete facing, spillway crest and apron are in fair condition with evidence of minor erosion on the crest and apron. Observations of the piers revealed minor hairline cracking, checking, efflourescence and surface staining due to rusting of the service bridge grid deck. Structural steel supports for the service bridge and sluice gate yokes are in good condition with occasional minor rusting. The submerged condition of the gates precluded visual inspection.

(3) Spillway with Stoplogs

The applicable comments relating to the stone foundation in the preceding paragraph apply to this structure. Due to tailwater conditions, the extent of seepage, if any, through the foundation could not be observed.

The concrete apron is in fair condition with evidence of minor surface erosion. Observation of the piers also revealed minor hairline cracking, checking, efflourescence and surface staining due to rusting of the service bridge grid deck. Structural steel supports for the service bridge and stoplogs are in good condition with occasional minor rusting. Stoplogs show no evidence of displacement or deflection under hydrostatic pressures.

(4) Spillway Structure

The applicable comments relating to the stone foundation again apply to this structure. Due to tailwater conditions, the extent of seepage, if any, through the foundation could not be observed. The concrete spillway is in fair condition with minor evidence of surface erosion. Minor joint erosion exists between the lower end of the spillway and the left side of the intermediate pier. Observations of the dividing piers revealed minor hairline cracking, checking, efflourescence and surface staining due to rusting of service bridge grid deck.

(5) Intermediate Pier

The intermediate pier is in fair condition. There is minor erosion at the outer end of the right corner of the structure. There is also evidence of minor checking and efflourescence on the left and right walls.

(6) Five-Bay Sluice Gate Structure (Photo 1)

The concrete in this structure, including the intermediate piers aprons and energy dissipator are in good condition with no evidence of erosion, spalls, cracks, checking or efflourescence. The sluice gates themselves are in good condition.

(7) Right Abutment (Photo 3)

The concrete portions of the right abutment, upstream wingwalls and downstream training walls are in good condition without evidence of significant erosion, spalls, cracks, checking or efflourescence.

Considerable erosion of the downstream channel bank near the end of the right training wall has been arrested by the placement of large boulders along the bank for a distance of 25 feet beyond the end of wall.

(d) Appurtenant Structures

(1) Service Building

The service building is in good condition with no significant structural defects. Despite the fact that the walls are not waterproofed, there is no evidence of moisture seepage through the walls or roof. New maintenance stoplogs for the five-bay sluice gate structure, which are stored in the building, are incorrectly fabricated. Slots at anchor bolt locations are deeply skived, reducing the cross sectional bearing area by as much as 50%.

(2) Service Bridges and Railings

The service bridge grid deck located between the left bank and intermediate pier suffers from minor surface corrosion. Structural supports and connections exhibit a minor degree of corrosion.

The service bridge grid deck spanning over the five-bay sluice gate structure is in good condition with no evidence of rusting or corrosion.

The pipe rail fence on both service bridges is in good condition without evidence of rusting or corrosion.

(3) Gate Operating Mechanisms

Operation of the gates at the sluice gate and spillway structure and the 5 bay sluice gate structure is either manual, utilizing a hand crank, or by a hand-held, electric powered portable operator similar to an electric drill. The portable electrical operator utilizes an extension cord plugged into an electrical receptacle at the service building.

The representative of the New Hampshire Water Resources Board advises that they limit the extent of gate opening by physically measuring the change in stem position. When not in use, the portable operator is stored in the service building.

It was found that the hand crank with its keyway and the power drive shaft with its keyway were not compatible for hand crank operation. Thus, none of the gates at either structure were operated on a purely manual basis using the hand crank.

During the operational testing of the gates at both sluice gate structures, the gates were operated at low speed during the initial opening and on approaching the full open position and at high speed in between. The reverse approach was used on closing.

At the time of inspection, the structural supporting members and the operating gate mechanisms were in good condition. All gates were individually raised and lowered utilizing the portable electrical operator and all operated satisfactorily. None of the gates were operated utilizing the hand crank due to the incompatibility of the keyway on the hand crank and the keyway of the shaft on the bench stand.

(e) Reservoir

An inspection of the reservoir shore revealed no evidence of movement or other instability. No significant sedimentation was observed behind the spillway or in the immediate upstream channel. Observation of the surrounding area revealed no work in progress or recently completed which might increase the flow of sediment into the reservoir. Additionally, there are no major changes to the surrounding watershed which might adversely affect the runoff characteristics of the basin.

(f) Downstream Channel

There are no downstream conditions which adversely affect the operation of the dam or which pose a hazard to the safety of the dam.

3.2 Evaluation

Because this dam is of basically straightforward design and because most of its major components are accessible for observation, the visual inspection permitted an overall satisfactory evaluation of those items which affect the safety of the structure.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures

As mentioned previously, the NHWRB's dam tender visits the dam at least every other day and reports gage readings back to the Board's engineering section. The engineering section, in turn, directs any operations deemed necessary. The Board draws this dam down two feet in the late summer or early fall in anticipation of fall storms and spring runoff.

4.2 Maintenance of Dam

The dam operator also inspects the condition of the dam during his visits and periodically files a written report with the Board. The engineering section then initiates whatever actions are necessary to effect repairs. Additionally, engineers from the Board inspect the dam periodically.

4.3 Maintenance of Operating Facilities

The procedures outlined in section 4.2 also apply to all operating facilities.

4.4 Description of Any Warning System in Effect

No formal warning system exists for this structure.

4.5 Evaluation

The operation and maintenance of this dam are well organized and accomplished satisfactorily. Because of the dam's hazard potential classification, the lack of a formal, written flood and emergency warning system is a significant shorcoming.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 Evaluation of Features

(a) Design Data

The primary source of data on Lochmere Dam is the files of the New Hampshire Water Resources Board (NHWRB). The files contain design drawings for the dam at various points in time as repair and modification projects have occurred. The most recent modifications occurred in 1976 and included the removal of the old headworks to the power canal and the canal itself. The headworks were replaced by a concrete structure containing five 6 foot by 6 foot underflow sluice gates. A rating curve for a single new gate was obtained from NHWRB. An independent analysis of the gates yielded a flow approximately fifteen percent greater than that given on the rating curve. Given that the rating curve may have been based on more detailed information and that it represents the more conservative of the estimates, it was accepted as valid and incorporated into the total rating curve for the dam.

No design flows were found in the NHWRB file, but an analysis by Fenton G. Keyes Associates for the Corps of Engineers in 1957 rated the capacity of the dam as 5,600 cfs. However, this analysis did not include the recently constructed sluice gates discussed above.

(b) Experience Data

Experience data for Lochmere Dam is discussed in subparagraph 1.3(b) (2) above.

(c) Visual Observations

The dam is well maintained and operated on a continual basis by the NHWRB. There is relatively little freeboard above the spillway crest. The structure supporting the gear drives for the sluice gates and the walkway have only about 2.5 feet of clearance between them and the spillway crest. The overbank on the right side will be overtopped when the head above the spillway exceeds 3 feet. An energy dissipator was constructed downstream of the five new sluice gates, but it should not create enough backwater to limit flow from the sluice gates.

(d) <u>Overtopping Potential</u>

The hydraulic conditions of interest in this Phase I investigation are those required to assess the adequacy of the dam in terms of its overtopping potential and its ability to safely allow an appropriately large flood to pass. This includes the determination of a Test Flood (TF) and a comparison of that peak flow to the discharge and storage capacities of the structure.

The Corps of Engineers' "Recommended Guidelines" for the Dam Safety Inspection Program provides guidance on the selection of a Test Flood based on the hazard and size classifications of the structure. For a structure classified as INTERMEDIATE in size and SIGNIFICANT in hazard, the recommended TF inflow to the reservoir above the dam is 1/2 PMF to PMF, where PMF is the Probable Maximum Flood. For New England, a PMF resulting from 19" of runoff is assumed. A chart of "Maximum Probable Flood Peak Flow Rates" as a function of drainage area and general topography was provided by the New England Division, Corps of Engineers.

The "Recommended Guidelines" suggest that where a range of test floods is indicated, the magnitude that most closely relates to the involved risk should be selected. On this basis, since the risk is considered to be on the lower end of the SIGNIFICANT category, a Test Flood based on the 1/2 PMF was selected.

Lochmere Dam is part of a complex hydraulic and hydrologic system and cannot be directly assigned a 1/2 PMF without consideration of the interactions between the various dams and reservoirs that comprise the total Winnipesaukee River system. The drainage area at Lochmere Dam is 428 square miles, but of that total, 374 square miles is located above Avery Dam in Laconia, 363 square miles above Lakeport Dam at Lakeport and 351 square miles above the narrow channel outflow of Lake Winnipesaukee at the Weirs. The surface area of Lake Winnipesaukee is 76 square miles and thus represents 22 percent of the discharge area above the Weirs. Immediately upstream of Lochmere Dam is Winnisquam Lake with a surface area of 6.5 square miles, or 12 percent of the incremental drainage area between Avery and Lochmere Dams. If a storm of the magnitude of a 1/2 PMF (assuming 10 inches of runoff) were to occur, two distinct peaks could be expected at Lochmere Dam.

Case A would be the primary peak representing runoff from the area immediately upstream of Lochmere Dam and not controlled by any other structures. Based on an incremental area of 54 square miles, the PMF runoff is taken from the COE curve to be roughly 1000 csm. Thus, the 1/2 PMF peak inflow may be calculated to equal 27,000 cfs. Assuming that Winnisquam Lake is at its normal elevation of 482 feet at the start of the storm the resulting peak outflow was determined to be approximately 9450 cfs. Again, this estimate is based on runoff from the 54 square miles between Lochmere and Avery Dams only.

Case B considers the secondary peak that would occur when the peak outflow from Lake Winnipesaukee reached Lochmere Dam. If 10 inches of runoff were to flow into Lake Winnipesaukee and no outflow or spreading of the surface area is considered, the maximum rise in the lake would be 3.8 feet. Based on rating information at the Weirs, a 3.8 foot rise would result in a maximum outflow from the lake of approximately 5000 cfs. This peak would coincide with the falling side of the runoff hydrograph from the 77 square miles located between the Weirs and Lochmere Dam. A net assumed runoff of 100 csm was assigned to the incremental area and then added to the peak discharge from Lake Winnipesaukee. The resulting estimate of the 1/2 PMF at Lochmere is then 12,700 cfs.

A more complete analysis of the system would include assumptions on rainfall distribution, storage and routing characteristics of each constriction between the Weirs and Lochmere. However, for the Phase I report it was concluded that this degree of analysis was not warranted. Given the two estimates and the assumptions associated with each, a Test Flood of 10,000 cfs was assigned to Lochmere Dam.

A peak of 10,000 cfs at Lochmere Dam exceeds the capacity of the dam. The dam's capacity without any overtopping of the walkway or overbanks is approximately 6000 cfs, assuming that the sluice gates are wide open, but that the stoplogs (3 feet assumed) have not been removed. The 10,000 cfs flow would result in the walkway and west overbank being overtopped by approximately 2 feet. If the stoplogs in 12 of the 6 foot wide bays were completely removed, the capacity of the dam would be increased by approximately 1850 cfs at H = 13, or a water surface elevation of 484.3 feet at the crest. Thus, the maximum capacity with the stoplogs removed is approximately 7800 cfs, which is still less than the test flood of 10,000 cfs.

5.2 Hydrologic/Hydraulic Evaluation

The results of the hydrologic and hydraulic assessment indicate that the dam has a greater capacity than any historic flood and that the existence of major lakes upstream will significantly lower any runoff peak flows, but that the dam would still be overtopped for a flood of the magnitude of a 1/2 PMF.

An area of concern is the lack of any significant freeboard on the right overbank. The area that was recently regarded as part of the construction of the new sluice gates would be overtopped and probably severely eroded during a major flood.

5.3 Downstream Dam Failure Hazard Evaluation

The flood hazards in downstream areas that would result from a failure of the dam were estimated using the procedure set forth in "Rule of Thumb Guidelines for Estimating Downstream Dam Failure Hydrographs," Corps of Engineers, New England Division, April 1978.

The assumed failure condition is that the water surface is at the spillway crest with the underflow sluice gates open full. Thus, there would be a flow of approximately 3000 cfs prior to failure. The estimated peak flow from a failure which opens an 80 foot wide gap in the dam would be 4250 cfs. Thus, the peak used to estimate downstream damages was set at approximately 7000 cfs.

This flow corresponds fairly closely with the estimated 500-year flood flow used in the Flood Insurance Studies for Tilton and Northfield, New Hampshire. An examination of the flood hazard maps prepared for those communities using surveyed cross sections and the HEC-2 program indicates that a flow of 7000 cfs would cause significant flooding in three locations. The locations are the southwest shore of Silver Lake where several cottages would be flooded, upstream of the Route 140 bridge where there are at least three low-lying structures, and in the central section of Tilton and Northfield, behind the Tilton Dam, where there are several older mills and commercial buildings immediately adjacent to the river. Property damage in all three areas would be a greater concern than loss of life given the expected levels of flooding and the close proximity of all three areas to safer high ground.

SECTION 6 - STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

(a) Visual Observations

The extensive field investigation of this dam do not reveal any displacement and/or distress which would warrant the preparation of structural stability calculations based on assumed sectional properties and technical values.

(b) Design and Construction Data

There are no design data available for review of the structural stability of the dam. While the existing construction drawings would provide some guidance in performing such calculations, the lack of foundation information and accurate data concerning the submerged portions of the dam would significantly decrease the value of any stability analysis.

(c) Operating Records

The operating records for the Lochmere Dam reveal no evidence of instability during historic peak flow periods.

(d) Post Construction Changes

The alterations to the original dam accomplished by the previous owner and by the NHWRB would be expected to increase the overall stability of the structure by providing additional weight and by permitting greatly increased discharge capabilities.

(e) Seismic Stability

The dam is located in Seismic Zone No. 2 and, in accordance with the recommended Phase I guidelines, does not warrant seismic analyses.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 Dam Assessment

(a) Condition

The Lochmere Dam is in GOOD condition at the present time.

(b) Adequacy of Information

The lack of in-depth engineering data does not permit a definitive review. Therefore, the adequacy of this dam cannot be assessed from the standpoint of reviewing design and construction data. The assessment is thus based primarily on the visual inspection, past performance history and sound engineering judgement.

(c) Urgency

The improvements described herein should be implemented by the owner within 2 years of receipt of the Phase I Inspection Report.

(d) Need for Additional Investigation

Since the discharge capacity of the dam is insufficient to pass the selected Test Flood and since the following subparagraph will recommend an improvement in this capacity, a refined hydrologic analysis to develop the appropriate Test Flood in a more detailed manner appears warranted.

7.2 Recommendations

Since the discharge capacity of the Lochmere Dam is insufficient to pass a 10,000 cfs Test Flood without overtopping, an engineering investigation to develop increased discharge capacity at the dam is recommended.

Additionally, a technical inspection of the dam should be conducted every two years.

7.3 Remedial Measures

The Lochmere Dam requires the following operating and maintenance improvements:

(1) Provide compatible hand cranks and keyways for all stems so that the gates may be operated in emergency situations when power is not available.

(2) Monitor erosion at the end of the right training wall and improve the rock slope protection if necessary.

(3) Monitor seepage through squared stone masonry near the left side sluice gates, noting particularly any changes in quantity. If the situation presents itself, conduct a detailed inspection of these areas under drawn down or other low water conditions.

(4) Install a gauge at the dam site to better monitor flow.

(5) Replace the inadequate maintenance stoplogs for the five bay sluiceway.

(6) Instruct local officials such as the police and fire chiefs in the proper operation of the dam and arrange for their access to operating equipment in the event of an emergency. Such a program might decrease response time in the event of unforseen circumstances.

(7) Institute a formal, written flood and emergency warning system.

7.4 Alternatives

As an alternative to an improvement of the dam's discharge capacity, the structure could be left as is with the potential for flooding in the event of a Test Flood magnitude storm. Since the storm of record in this area, which occurred in 1938, is less than 40 percent of the Test Flood, this alternative may be a viable one.

There are no meaningful alternatives to the operating and maintenance type improvements.

APPENDIX A

VISUAL INSPECTION CHECKLIST

INSPECTION TEAM ORGANIZATION

Date: 31 May 1978

NH 00015
LOCHMERE DAM
Belmont, New Hampshire
Winnipesaukee River
NHWRB 21.07

Weather: Sunny and warm

INSPECTION TEAM

James H. Reynolds	Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZD)	Team Captain
William S. Zoino	GZD	Soils
Nicholas Campagna	GZD	Soils
Andrew Christo	Andrew Christo Engineers (ACE)	Structural
Paul Razgha	ACE	Structural
David Duncan	Bethel, Duncan and O'Rourke, Inc.	Mechanical
Guillermo Vicens	Resource Analysis, Inc.	Hydrology

Mr. Robert Vay, dam tender for the NHWRB, accompanied the inspection team.

CHECK LISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
DAM SUPERSTRUCTURE		
a. General		
Settlement or movement of crest	nac	None noted
Vertical alignment		No deficiencies noted
Horizontal alignment		No deficiencies noted
b. Condition at abutments		
Settlement or movement of crest		None noted
Trespassing on slopes		None noted
Sloughing or erosion of slopes		Large eroded area at end of right training wall; erosion arrested by placement of heavy boulders
Rock slope protection		No deficiencies noted on left side; right side as mentioned above
Unusual movement or cracking at or near toes		None noted
Unusual embankment or downstream seepage		None noted
Piping or boils		None noted
Foundation drainage features		Unknown
Toe drains	nac	Unknown

CHECK LISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
OUTLET WORKS		
a. Approach Channel		
Slope conditions	nac	No evidence of instability
Bottom conditions		Deep approach
Log boom		None
Debris		None noted
Trees overhanging channel	nac	None
b. Five Bay Sluice Gate Structure	PE	
General condition of concrete		Good
Rusting or staining		None
Spalling		None
Erosion or cavitation		None
Visible reinforcing		None
Seepage or efflourescence		None
Cracking		None
Junctions with pier and right abutment		No deficiencies noted
Condition of sluice gates	PE	Good

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
Condition of operating mechanisms	PR	Good, but hand/crank does not fit stem; all gates properly operated with portable electric operator; maintenance stoplogs improperly fabricated, thus in poor condition
c. Spillway Structures		
General condition of concrete, spillway caps and apron		Concrete in good condition; minor surface erosion on both structures; minor joint erosion on spillway structure
Squared stone masonry		Good condition; seepage, if any exists, could not be observed
Intermediate concrete piers		Minor hairline cracking, checking, efflourescence and rust staining (from steel service bridge)
Condition of stoplogs		Good
Junction with pier		No deficiencies noted
d. Sluice Gate and Spillway structure		
General condition of concrete spillway cap and aprons		Concrete in good condition; minor surface erosion
Squared stone masonry	PR	Good condition; seepage observed when sluice gates operated

CHECK LISTS FOR VISUAL INSPECTION

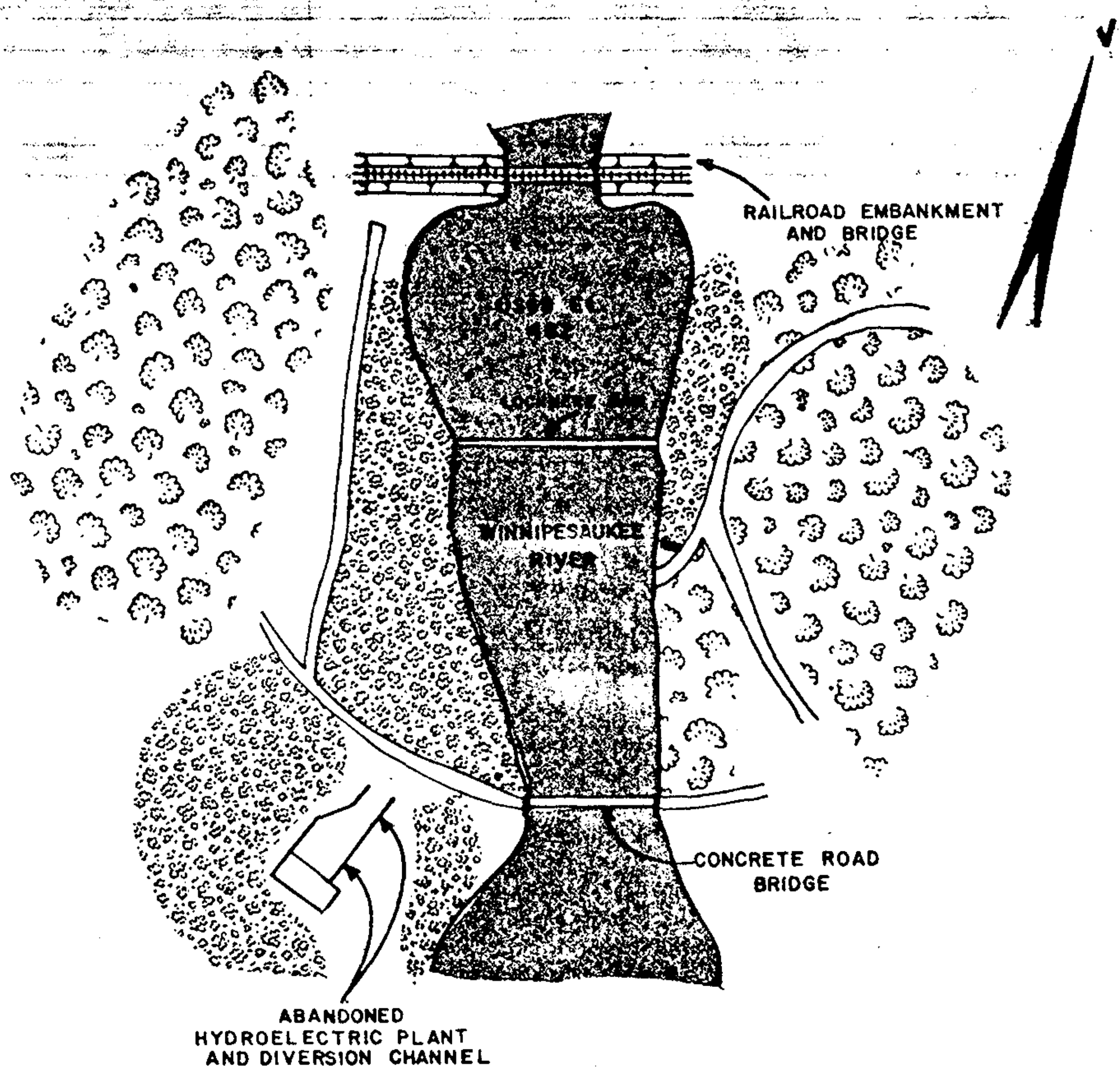
AREA EVALUATED	BY	CONDITION & REMARKS
Intermediate concrete piers	PP	Minor hairline cracking, checking, efflourescence and rust staining (from steel service bridge)
Condition of operating mechanisms		Good, but manual key does not fit stem; all gates operated with portable electric operator
Condition of gates		Not observed due to submerged condition
Junction with left abutment		No deficiencies noted
e. Service building and bridge		
Bearing		Minor corrosion on bridge supports
General condition of concrete and masonry		Good
Expansion joints		No deficiences noted
Metal grid deck		Good condition
Handrails		Good condition
f. Outlet Channel	PP	
Condition of concrete aprons		Minor surface erosion
Trees overhanging channel	nac	None of significance
Floor of channel	nac	Rocky

CHECK LISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
Other obstructions	nac	Deteriorated road bridge 700 feet downstream could wash out and become obstruction due to its condition
g. Existence of gages		USGS gages upstream and downstream; none at dam itself
RESERVOIR		
a. Shoreline		
Evidence of slides		None
Potential for slides		Shoreline stable
b. Sedimentation		None noted
c. Upstream hazard areas in the event of back-flooding		Many residences and businesses around Lake Winnisquam
d. Changes in nature of watershed (agriculture, logging, construction, etc.)		None noted
DOWNSTREAM CHANNEL		
Restraints on dam operation		None noted
Potential flooded areas	nac	Some low lying areas around Silver Lake and through Tilton

CHECK LISTS FOR VISUAL INSPECTION		
AREA EVALUATED	BY	CONDITION & REMARKS
OPERATION AND MAINTANANCE FEATURES		
a. Reservoir regulation plan		
Normal procedures	mac	Regulate Lake Winnisquam for recreational purposes
Emergency procedures		Minimize flooding along Winnipesaukee River
Compliance with designated plan		Satisfactory
b. Maintenance		
Quality		Satisfactory
Adequacy	mac	Satisfactory

APPENDIX B



GOLDBERG, ZOINO, DUNNICLIFF & ASSOC, INC. GEOTECHNICAL CONSULTANTS NEWTON UPPER FALLS, MASS.	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

FIGURE I

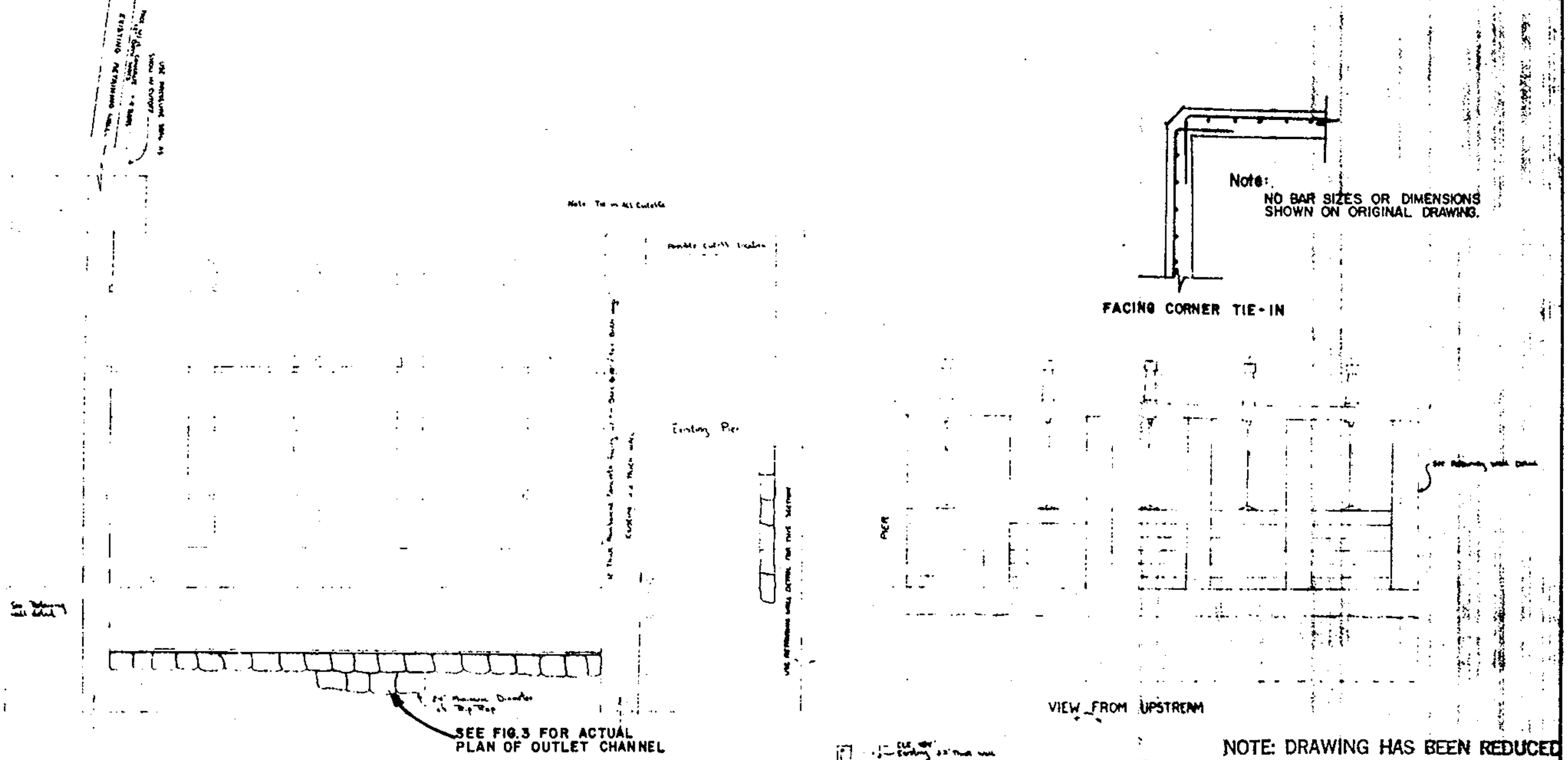
SITE PLAN

LOCHMERE DAM — NEW HAMPSHIRE

SCALE 1/2" = 100'

DATE SEPT. 1978

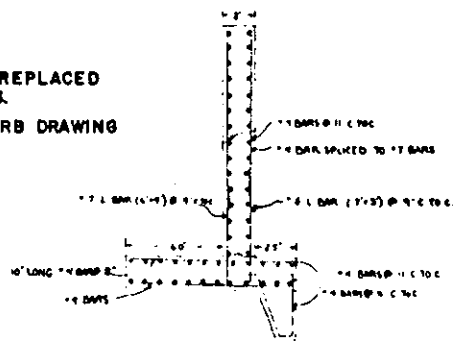
FILE No. 2067



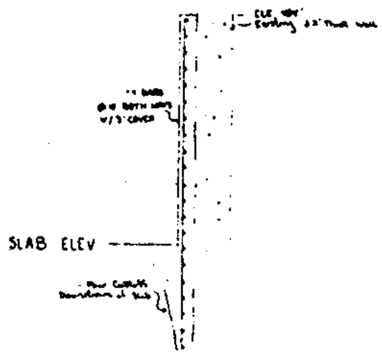
NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN

NOTES

1. EXISTING PIER ALSO REPLACED DURING MODIFICATIONS.
2. ADAPTED FROM NHWRB DRAWING DATED 10/4/76



RETAINING WALL



PIER FACING DETAIL

GOLDBERG, ZOINO, DUNNICLIFF & ASSOCIATES
GEOTECHNICAL CONSULTANTS
NEWTON UPPER FALLS, MASS.

U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS

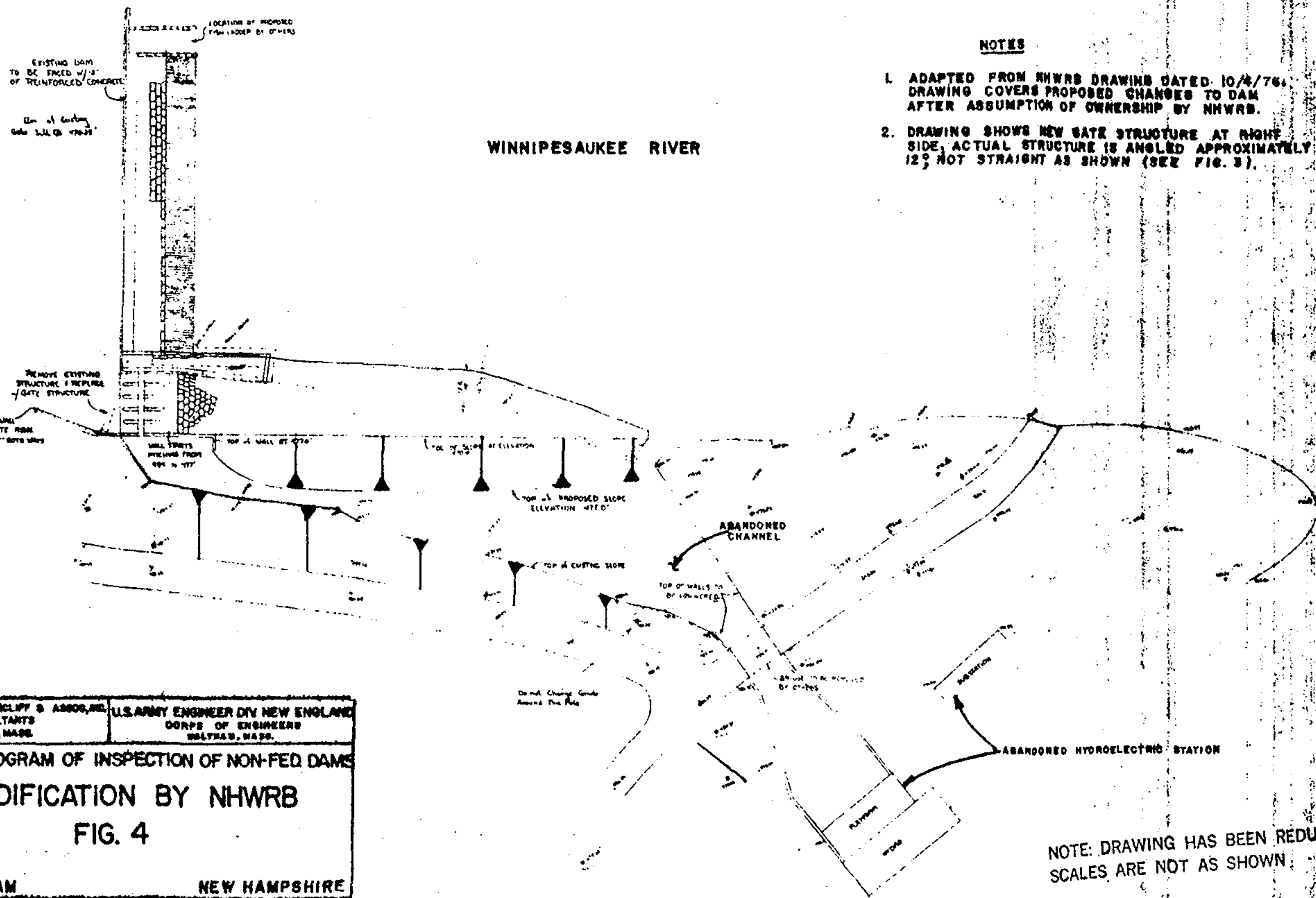
NEW RIGHT GATE STRUCTURE

FIG. 3

LOCHMERE DAM — NEW HAMPSHIRE

FILE No. 2067

SCALE	NO SCALE
DATE	SEPT 1978



NOTES

1. ADAPTED FROM NHWRB DRAWING DATED 10/4/76. DRAWING COVERS PROPOSED CHANGES TO DAM AFTER ASSUMPTION OF OWNERSHIP BY NHWRB.
2. DRAWING SHOWS NEW GATE STRUCTURE AT RIGHT SIDE; ACTUAL STRUCTURE IS ANGLED APPROXIMATELY 12°, NOT STRAIGHT AS SHOWN (SEE FIG. 3).

NOTE: DRAWING HAS BEEN REDUCED SCALES ARE NOT AS SHOWN

...DBERG, ZOINO, DUNNICLIFF & ASSOC, INC. ...TECHNICAL CONSULTANTS ...TON UPPER FALLS, MASS.	U.S. ARMY ENGINEER DIV NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

...TIONAL PROGRAM OF INSPECTION OF NON-FED DAMS

MODIFICATION BY NHWRB

FIG. 4

...CHMERE DAM — NEW HAMPSHIRE

SCALE: NO SCALE
DATE: SEPT 1978

The NHWRB, 37 Pleasant Street, Concord, N.H. 03301 maintains the following documents concerning this dam:

(a) Operational records of the dam since the advent of state ownership in 1966 plus any records turned over by the previous owner.

(b) Four pages of hand-drawn sketches relating to the modifications accomplished in 1977.

(c) An undated discharge curve for the new right gate structure.

(d) A 1938 report by the New Hampshire Water Control Commission entitled "Data on Dams in New Hampshire."

(e) A 1938 report by the same agency entitled "Data on Water Power Developments in New Hampshire."

(f) Two reports, one prepared by the Department of Housing and Urban Development and one prepared by the Corps of Engineers concerning the hydrology of the Winnipesaukee River.

The Board's telephone numbers are (603) 271-3406 or (603) 271-1110.

Location & Project No. Lochmere Date March 1974

Boring No. 1 — Ground

Elev.

Description	Blows
Sand + Stones Fill	16
	12
	11
	6
(H_2O at 6)	7
Silt	0
	0
	12
Sand	14
	25
	54
Sandy Till + Bldrs	74
	72
	90
	132
	360
	145
	269
	429
	85
	369
	411
	511
	140
	423
	631
Bott. of Hole	

Boring No. 2 — Ground

Elev.

Description	Blows
Sand + Stones Fill	1
	7
	196
	8
(H_2O at 6)	10
	23
wood	61
	44
	147
Sandy Till + Bldrs.	547
	614
	135
	144
	281
	396
	442
	987
	211
	571
	562
	331
	142
	790
	[illegible]
Bott. of Hole	

Boring No. 3 — Ground.

Elev.

Description	Blows
Sandy + Stonel Fill	3
	3
	4
	6
(H_2O at 6)	19
	22
Sandy Till + Bldrs	65
	176
	104
	114
	391
	107
	156
	396
	146
	230
	571
	122
	741
	102
	316
	712
	145
	396
	943
	164
	536
	141
	[illegible]
Bott. of Hole	

Boring No. 4 — Stream Bed

Elev.

Description	Blows
Sandy Till + Bldrs	16
	18
	125
	31
	35
	76
	175
	270
	402
	629
Bott. of Hole	

Figures in right hand column indicate number of blows required to drive 1 5/8" O.D. A-rod one foot, using 140 lb. weight falling 30 inches ....

Signed

Location & Project No. Lochmere Date. March 1976

Boring No. 5 — Stream Bed

Elev.........

Depth	Description	Blows
2	Sandy Till	9
		10
4	& Bldrs.	33
		50
6		
		110
8		96
		244
10		310
		321
12	Bott. of Hole	

Boring No. 6 — Stream Bed

Elev.........

Depth	Description	Blows
2	Sandy Till	33
		37
4	& Bldrs.	38
		114
6		
		246
8		139
		444
10		527
		1121
12	Bott. of Hole	

Boring No. 7 — Stream Bed

Elev.........

Depth	Description	Blows
2	Sandy Till	23
		41
4	& Bldrs.	38
		363
6		410
		341
8		404
		777
10	Bott. of Hole	

Boring No. ......

Elev.........

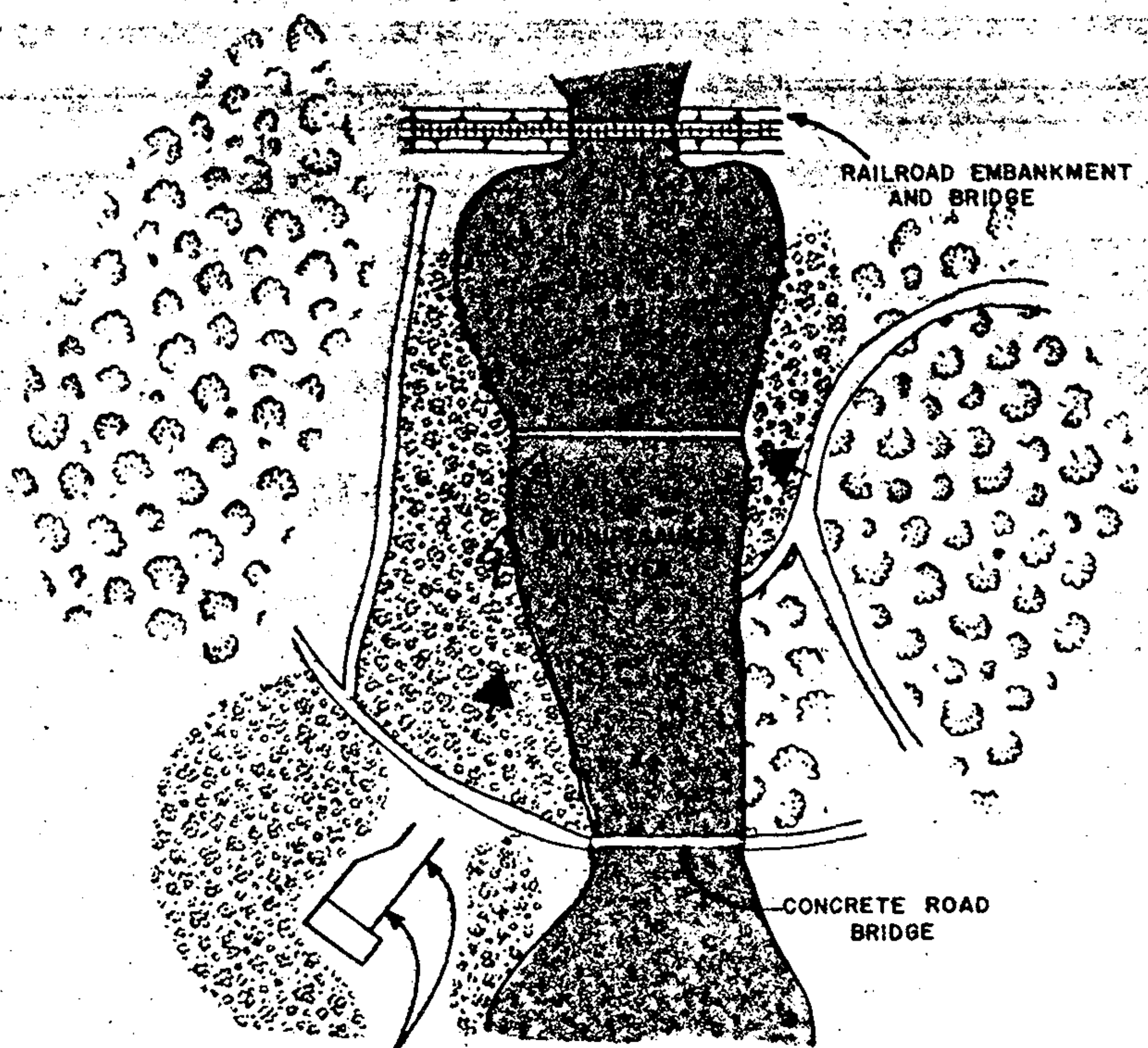
Depth scale: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34, 36, 38, 40, 42, 44, 46, 48, 50

Figures in right hand column indicate number of blows required to drive 1 5/8" O.D. A-rod one foot, using 140 lb. weight falling 30 inches ....

Signed

APPENDIX C

SELECTED PHOTOGRAPHS



OVERVIEW PHOTOS

APPENDIX C PHOTOS

GOLDBERG, ZOINO, DUNNICLIFF & ASSOC, INC. GEOTECHNICAL CONSULTANTS NEWTON UPPER FALLS, MASS.	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCATION AND ORIENTATION OF PHOTOS

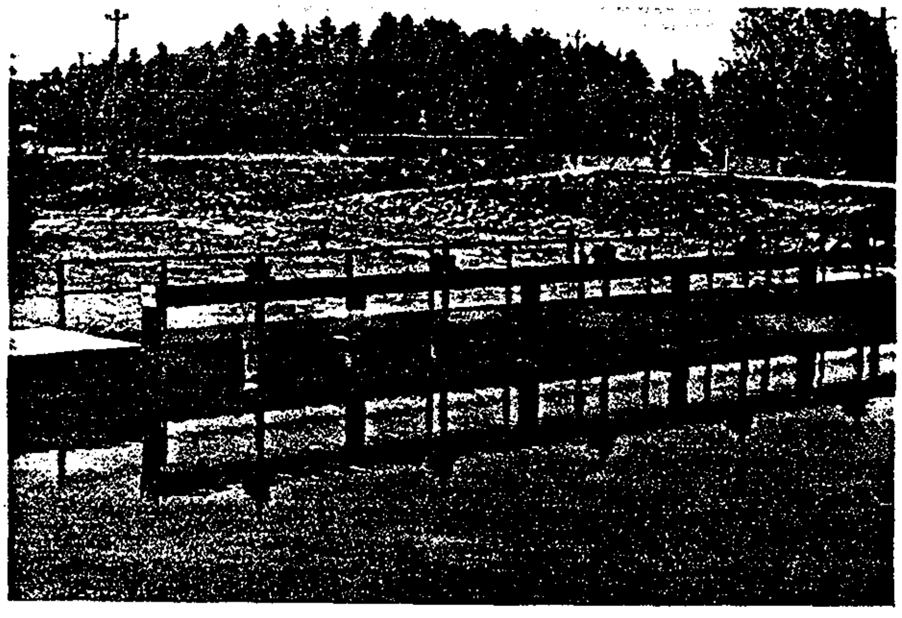
FILE No. 2067

LOCHMERE DAM — NEW HAMPSHIRE

SCALE 1/2" = 100'

DATE SEPT 1978

1. View from downstream of right side gate structure



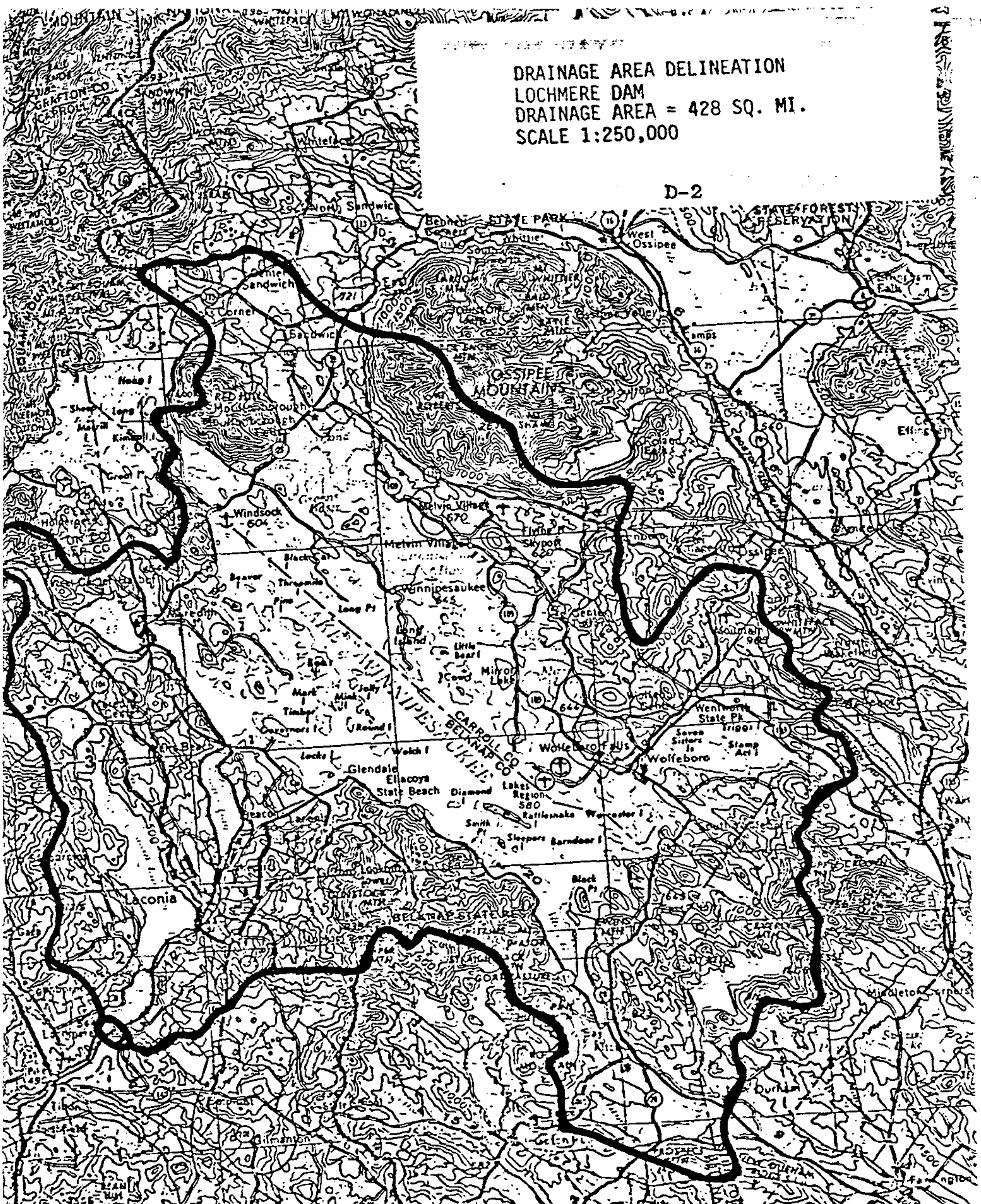
2. View from upstream of operating mechanisms for left side waste gates

3. View from downstream of repairs to eroded area near toe of right side outlet channel

APPENDIX D

HYDROLOGIC AND HYDRAULIC COMPUTATIONS

LOCHMERE DAM



DRAINAGE AREA DELINEATION
LOCHMERE DAM
DRAINAGE AREA = 428 SQ. MI.
SCALE 1:250,000

DAMS 148 : LOCHMERE DAM

SIZE CLASSIFICATION: INTERMEDIATE

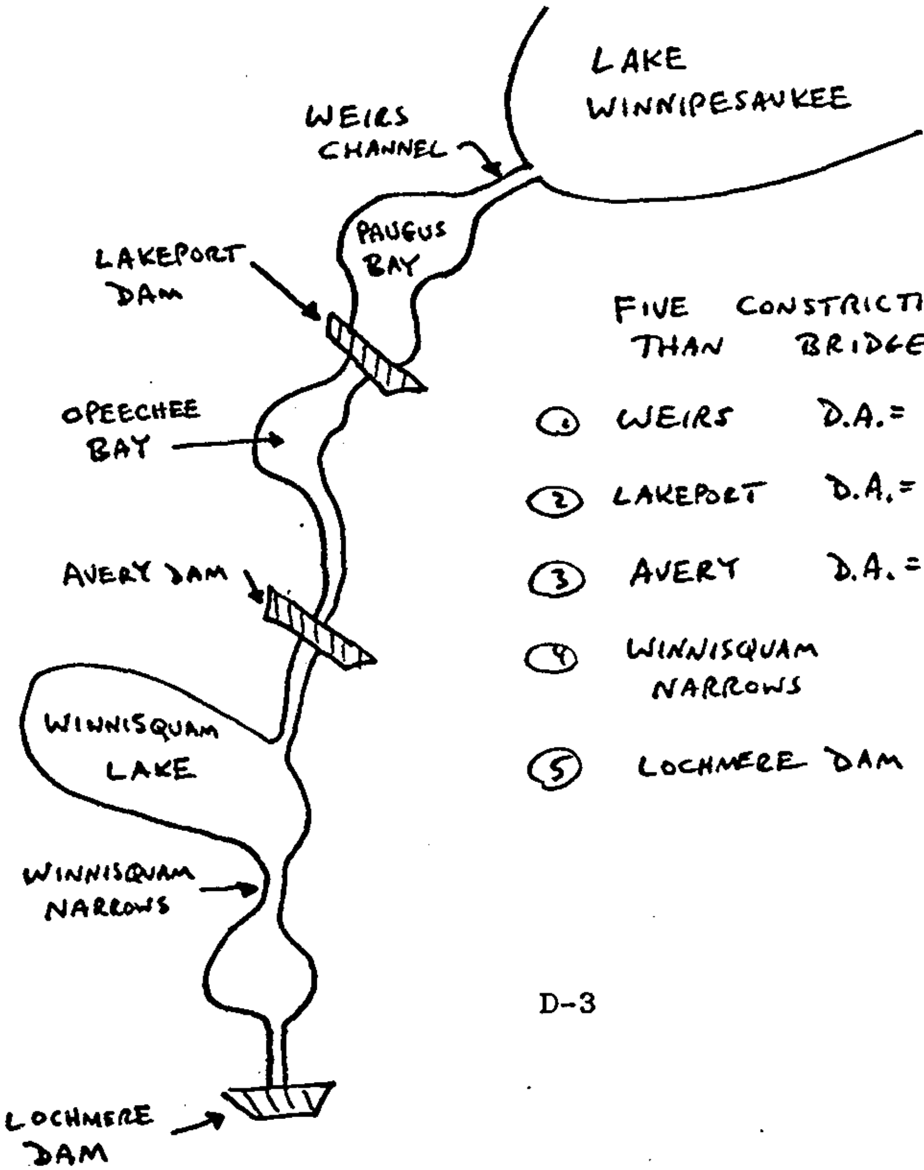
HAZARD CLASSIFICATION: SIGNIFICANT

COTTAGES AROUND SILVER LAKE DOWNSTREAM AND GENERAL DEVELOPMENT IN TILTON, N.H.

TEST FLOOD:

CoE GUIDELINES CALLS FOR ½ PMF → PMF

D.A. AT DAM = 428 SQMI BUT THERE ARE VARIOUS UPSTREAM CONTROLS. SEE SKETCH.



FIVE CONSTRICTIONS OTHER THAN BRIDGES:

1. WEIRS D.A. = 351 SQMI
2. LAKEPORT D.A. = 363 SQMI
3. AVERY D.A. = 374 SQMI
4. WINNISQUAM NARROWS
5. LOCHMERE DAM D.A. = 428 SQMI

THE SURFACE AREA OF LAKE WINNIPESAUKEE ≈ 72 SQ.MI. AT NORMAL ELEVATION (504 MSL). THIS REPRESENTS 72/351 = 21% OF D.A. UPSTREAM OF THE WEIRS. THE LARGE SURCHARGE STORAGE VOLUME AND CONSTRICTIONS AT WEIRS, LAKEPORT, AND AVERY WILL TEND TO DELAY ANY RUNOFF PEAK FROM THE LAKE UNTIL AFTER THE RUN OFF FROM THE AREA BETWEEN LOCHMERE & AVERY DAMS. THUS THE QUESTION THAT MUST BE ANSWERED IS WHETHER THE PMF OR ½ PMF AT LOCHMERE WOULD COME FROM LOCAL RUNOFF OR FROM THE ATTENUATED PEAK OUTFLOW OF LAKE WINNIPESAUKEE.

A FULL HYDROLOGIC ROUTING ANALYSIS OF VARIOUS STORM PATTERNS WOULD BE REQUIRED TO FULLY ANSWER THAT QUESTION. FOR THIS REPORT A SERIES OF ASSUMPTIONS MUST BE MADE. THE HAZARD RATING IS ON THE LOW SIDE OF THE SIGNIFICANT RANGE. THUS A RUNOFF OF 10" WILL BE ASSUMED. IF 10" OF RUNOFF WERE TO FLOW INTO LAKE WINNIPESAUKEE WITHOUT ALLOWING FOR OUTFLOW OR SPREADING THE LAKE WOULD RISE 4.1'. $\left[\frac{351}{72} \frac{mi^2}{mi^2} \times 10'' \times \frac{1'}{12''} \right]$. THIS RISE WHEN COMPARED TO THE RATING CURVE OF THE WEIRS DEVELOPED

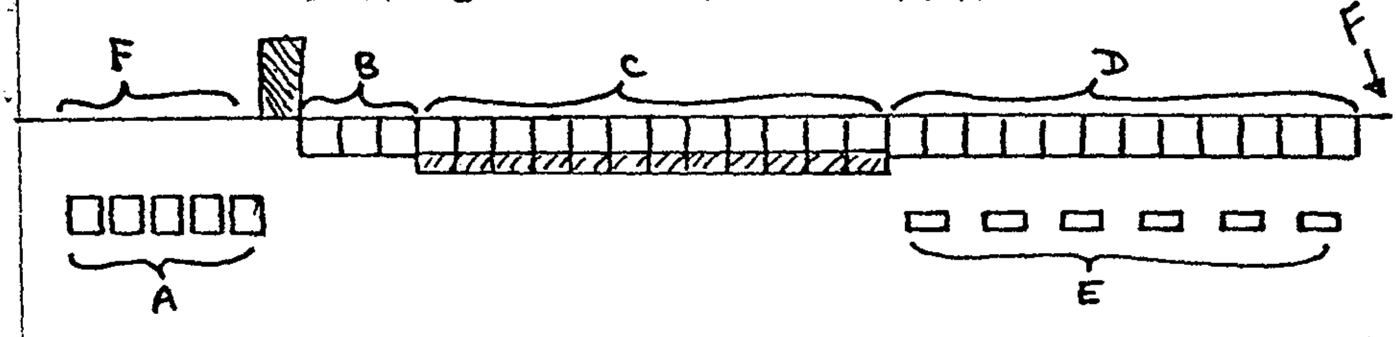
FOR THE LACONIA FLOOD INSURANCE STUDY YIELDS A PEAK DISCHARGE OF ≈ 5000 cfs. IF FROM THE FALLING SIDE OF THE STORM HYDROGRAPH WE ADD ANOTHER 100 CSM FOR THE AREA BETWEEN WEIRS AND LOCHMERE (77 SQMI) THIS WOULD YIELD A TOTAL POSSIBLE FLOW AT LOCHMERE OF 12,700 CFS, FOR THE PEAK THAT ~~COI~~ COINCIDES WITH THE LAKE WINNIPESAUKEE RUNOFF PEAK. ASSUMES WINNISQUAM IS HIGH AND THUS DOES NOT ATTENUATE PEAK.

AN EARLIER PEAK WOULD OCCUR WHEN THE PEAK FROM WINNISQUAM LAKE PASSES THE DAM. IF WE CONSIDER ONLY THE 54 SQMI BELOW THE AVERY DAM THE PMF FOR "ROLLING" TERRAIN WOULD YIELD ABOUT 1250 CSM FROM THE CURVE PROVIDED BY CoE. THE AREA AROUND WINNISQUAM LAKE WOULD PROBABLY FALL BELOW THE "ROLLING" CATEGORY AND THUS 1000 CSM WOULD BE A FAIR ESTIMATE OF THE INFLOW PMF TO WINNISQUAM LAKE.

$\frac{1}{2}$ PMF = $\frac{1}{2}$(54 mi^2 x 1000 csm) = 27,000 cfs.

DEVELOPMENT OF DISCHARGE/STAGE RATING CURVE

THE LOCHMERE DAM CONSISTS 5 TYPES OR LOCATIONS OF DISCHARGE POINTS, EXCLUDING OVERTOPPING OF THE DAM OR OVERBANKS. THEY ARE SHOWN SCHEMATICALLY BELOW WITH DESCRIPTIONS



A: FIVE DEEP SLUICE GATES. INVERT ELEVATION = 471.3*
MAX. OPENING OF EACH GATE = 6' x 6'.

B: THREE BAY SPILLWAY. INVERT ELEVATION = 481.3 MSL
THREE OPENINGS OF 5' 6".
OGEE SECTION.

C. TWELVE SLUICEWAY OPENINGS WITH STOPLOGS.
INVERT ELEVATION 477.9
ASSUMED 36" 15 INCHES (3.0' 1.25 FT) OF STOPLOG IN PLACE.
WIDTH OF EACH OPENING 5.6', ELEV TOP OF LOGS = 480.9

D. TWELVE ~~REL~~ SPILLWAY OPENINGS
INVERT ELEVATION 481.3
WIDTH OF EACH OPENING 5.6'

E. SIX DEEP SLUICE GATES. INVERT ELEVATION = 471.3
MAX. OPENING OF EACH GATE = 4'1"W x 2'9"H.

* ASSUMES NEW GATES HAVE SAME INVERT AS OLD GATES AS PER NHWRB.

NOTE: ELEVATIONS BASED ON SETTING OGEE SPILLWAY 481.3 = 96.5

"H" will BE DEFINED AS THE DISTANCE ABOVE THE SLUICE GATE SILLS (ELEV. 471.3').

FOR A MAJOR FLOOD IT WILL BE ASSUMED THAT ALL OF THE SLUICE GATES ARE OPENED FULL, BUT THAT THE STOP LOGS CAN NOT BE REMOVED.

THE LENGTHS OF THE VARIOUS WEIRS IN B, C, & D WILL BE SHORTENED IN THE COMPUTATION TO ACCOUNT FOR THE MANY PIERS, UNTIL THE WALKWAY ELEVATION IS EXCEEDED. THEN THE ENTIRE DAM LENGTH, LESS THE CONTROL BLDG WILL BE CONSIDERED AS ONE LONG WEIR.

FOR $H \leq 13.0'$ THE DISCHARGES ARE COMPUTED AS FOLLOWS:

$$Q_A = 5\left[C_d(A)\sqrt{2gH}\right]$$
$$= 5\left[.45(36)\sqrt{64.4(H)}\right]$$
$$= 81\sqrt{64.4H}$$

[ROUSE: ENGINEERING HYDRAULICS, PG 52
C_d SET AT 0.45 TO YIELD NHWRB DESIGN FLOW AT $H = 10.'$

Q_E: THE EXACT SHAPE OF THE TUNNELS LEADING FROM THE OLDER SLUICE GATES IS UNCERTAIN. ASSUME THEY DO ACT AS UNDERFLOW SLUICE GATES

$$\frac{b}{H} \approx \frac{2.75}{10} = .275 \Rightarrow C_d = .56 \quad \{\text{FROM ROUSE}$$

$$Q_E = 6\left[C_d(A)\sqrt{2gH}\right]$$
$$= 6\left[.56(4.08 \times 2.75)\sqrt{2(32.2)H}\right]$$
$$= 37.7\sqrt{64.4H}$$

NOTE: FROM ROUSE I WOULD HAVE PICKED ≈.52 BUT I FELT I SHOULD MATCH NHWRB AS LONG AS Δ WAS SMALL.

Q_B: THE EFFECTIVE SPILLWAY LENGTH IS DETERMINED FROM:

$$L = L_o - KNH_e$$

CHOW: OPEN-CHANNEL HYDRAULICS PG. 370

$$L = 16.5 - .02(6)(H-10)$$

$$Q_B = CLH^{3/2}$$

$$Q_B = 3.2\left[16.5 - .02(6)(H-10)\right]\left[H-10\right]^{3/2}$$

Q_C:

$$L = L_o - KNH_e$$

$$L = 66 - .02(24)(H-9.6)$$

$$Q_C = 3.0\left[66 - .02(24)(H-9.6)\right]\left[H-9.6\right]^{3/2}$$

Q_D:

$$L = L_o - KNH_e$$

$$L = 66 - .02(24)(H-10)$$

$$Q_D = 2.8\left[66 - .02(24)(H-10)\right]\left[H-10\right]^{3/2}$$

Q_F: ONCE THE WALKWAY ON THE NEW GATE ARE OVERTOPPED THAT LENGTH: 38 FT PLUS A SIGNIFICANT AREA ON THE LEFT OVERBANK (LOOKING UPSTREAM) - ASSUME 30 L.F., AND A SMALLER AREA ON THE RIGHT OVERBANK, ASSUME 12 L.F.

THUS $Q_F = 2.8 * 80 * (H-13)^{1.5}$

```
LIST
100 REMARK: DISCHARGE/STAGE CALCS FOR LOCHMERE DAM
110 PAGE
120 E=1.5
130 PRINT "DISCHARGE FROM LOCHMERE DAM"
140 PRINT USING 150:
150 IMAGE // 2T"HEAD"30T"DISCHARGE"
160 PRINT USING 170:
170 IMAGE 1T"(FEET)"32T"(CFS)"
180 PRINT USING 190:
190 IMAGE 10T"TOTAL     QA    QB     QC     QD     QE     QF"
200 FOR H=9 TO 18 STEP 0.25
210 Q1=81*(64.4*H)↑0.5
220 Q5=37.7*(64.4*H)↑0.5
230 Q7=0
240 Q2=0
250 Q3=0
260 Q4=0
270 IF H<=9.6 THEN 370
280 Q3=3*(66-0.02*24*(H-9.6))*(H-9.6)↑E
290 IF H<=10 THEN 370
300 Q2=3.2*(16.5-0.02*6*(H-10))*(H-10)↑E
310 Q4=2.8*(66-0.02*24*(H-10))*(H-10)↑E
320 IF H<=13 THEN 370
330 Q2=3.2*18*(H-10)↑E
340 Q3=3*72*(H-9.6)↑E
350 Q4=2.8*72*(H-10)↑E
360 Q7=2.8*80*(H-13)↑E
370 Q6=Q1+Q2+Q3+Q4+Q5+Q7
380 PRINT USING 390:H,Q6,Q1,Q2,Q3,Q4,Q5,Q7
390 IMAGE 1T,2D.2D,8D,8D,6D,7D,7D,7D,7D
400 NEXT H
410 END
```

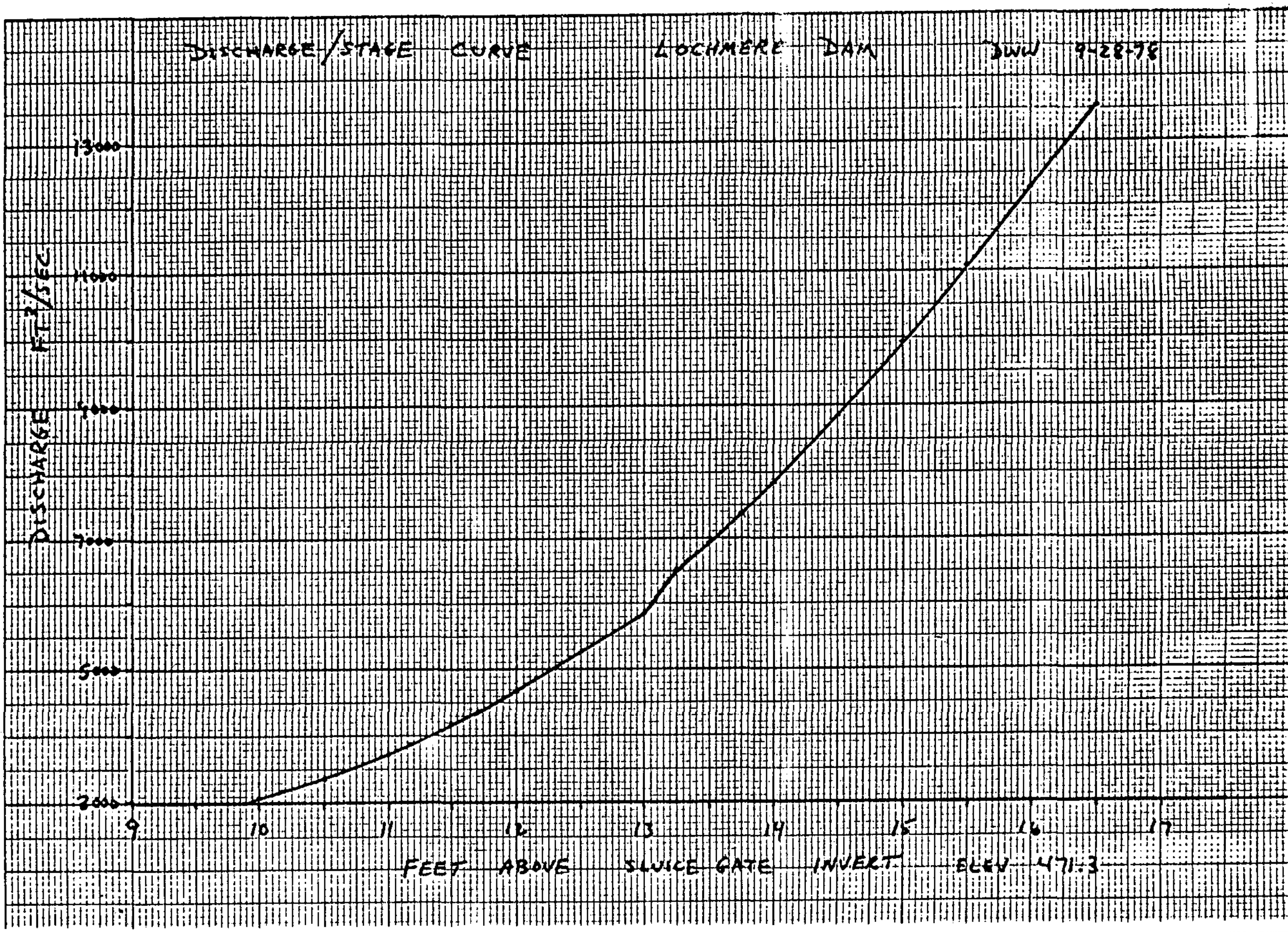
DISCHARGE FROM LOCHMERE DAM

HEAD (FEET)				DISCHARGE (CFS)			
	TOTAL	QA	QB	QC	QD	QE	QF
9.00	2858	1950	0	0	0	908	0
9.25	2897	1977	0	0	0	920	0
9.50	2936	2004	0	0	0	932	0
9.75	2986	2030	0	11	0	945	0
10.00	3062	2056	0	50	0	957	0
10.25	3183	2081	7	103	23	969	0
10.50	3338	2106	19	168	65	980	0
10.75	3519	2131	34	242	119	992	0
11.00	3720	2156	52	325	183	1003	0
11.25	3939	2180	73	415	256	1015	0
11.50	4173	2204	96	511	336	1026	0
11.75	4423	2228	121	614	422	1037	0
12.00	4685	2252	147	723	515	1048	0
12.25	4960	2275	175	838	613	1059	0
12.50	5247	2298	205	957	717	1070	0
12.75	5545	2321	236	1082	826	1080	0
13.00	5853	2344	268	1211	939	1091	0
13.25	6520	2366	337	1506	1181	1101	28
13.50	6940	2388	377	1664	1320	1112	79
13.75	7386	2410	418	1826	1464	1122	145
14.00	7855	2432	461	1994	1613	1132	224
14.25	8346	2454	505	2166	1766	1142	313
14.50	8856	2475	550	2343	1924	1152	412
14.75	9385	2496	596	2524	2087	1162	519
15.00	9931	2518	644	2710	2254	1172	634
15.25	10495	2538	693	2901	2425	1181	756
15.50	11075	2559	743	3096	2600	1191	885
15.75	11670	2580	794	3294	2780	1201	1022
16.00	12281	2600	847	3497	2963	1210	1164

16.25	12906	2620	900	3704	3150	1229	1312
16.50	13546	2640	955	3915	3341	1229	1467
16.75	14200	2660	1010	4130	3535	1238	1627
17.00	14868	2680	1067	4348	3734	1247	1792
17.25	15549	2700	1124	4570	3935	1257	1963
17.50	16243	2719	1183	4796	4141	1266	2138
17.75	16950	2739	1243	5026	4350	1275	2319
18.00	17669	2758	1303	5259	4562	1284	2504

9 of

D-12

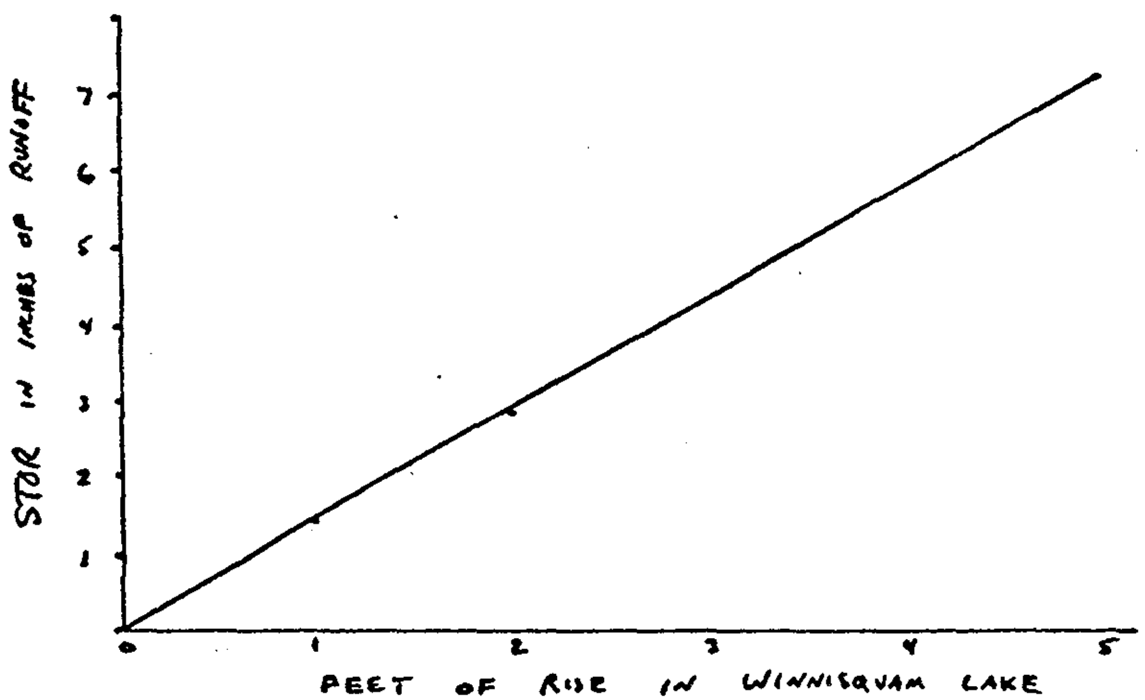


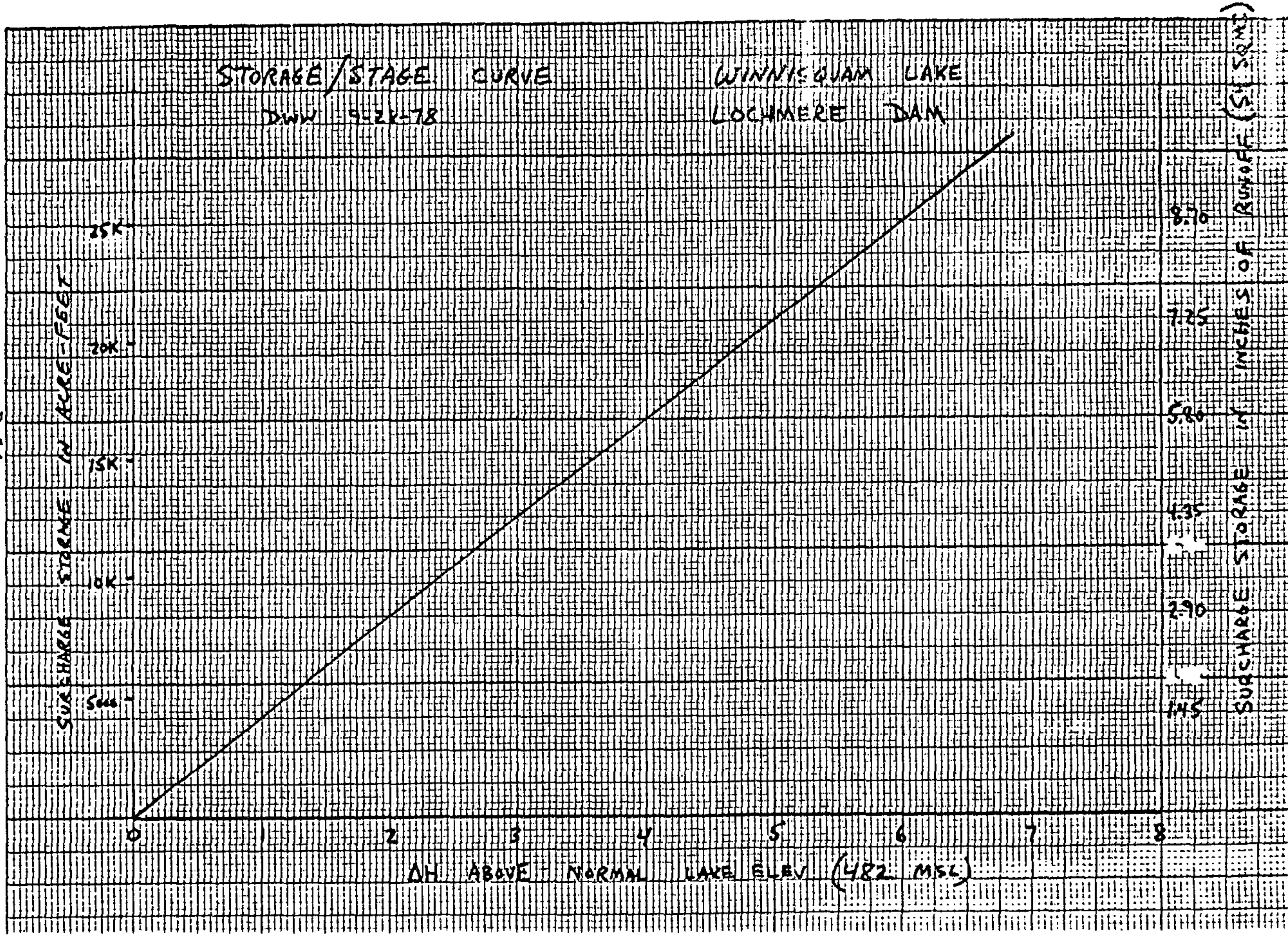
DISCHARGE/STAGE CURVE
LOCHMERE DAM
DWW 9-28-78
DISCHARGE FT³/SEC
13000
11000
9000
7000
5000
2000
9
10
11
12
13
14
15
16
17
FEET ABOVE SLUICE GATE INVERT ELEV 471.3

STORAGE / STAGE CURVE WILL BE BASED ON THE ASSUMPTION THAT THE NORMAL SURFACE AREA OF WINNISQUAM LAKE (ELEV 482) IS 6.5 SQMI, AND THAT THE SURCHARGE STORAGE IS DETERMINED BY MULTIPLYING THAT AREA BY THE HEAD ABOVE THE LOCHMERE SPILLWAY CREST. THE STORAGE ~~[illegible]~~N IN TERMS OF INCHES OF RUNOFF WILL ONLY CONSIDER THE 54 SQMI DRAINAGE AREA BETWEEN LOCHMERE + AVERY DAMS.

$$\frac{54 \text{ SQMI}}{6.5 \text{ SQMI}} \times 1'' = 8.3'' \text{ OF RISE ON LAKE FOR } 1'' \text{ OF RUNOFF}$$

$$1 \text{ FOOT OF RISE} = \frac{12}{8.3} = 1.45'' \text{ OF RUNOFF}$$





STORAGE/STAGE CURVE
DWW 9-26-78
WINNISQUAM LAKE
LOCHMERE DAM
SURCHARGE STORAGE IN ACRE-FEET
25K
20K
15K
10K
5000
SURCHARGE STORAGE IN INCHES OF RUNOFF (51 SQMI)
8.70
7.25
5.80
4.35
2.90
1.45
0
1
2
3
4
5
6
7
8
ΔH ABOVE NORMAL LAKE ELEV (482 MSL)

UCTION IN OUTFLOW DUE TO SURCHARGE STORAGE

ME TOTAL STORM VOLUME = 10" OF RUNOFF

$_2 = Q_{P1}\left(1 - \frac{STOR_1}{RUNOFF}\right)$

$_2 = 27000\left(1 - \frac{15.95}{10}\right)$

HEAD REQUIRED FOR 27000 CFS IS 11 FT ABOVE SPILLWAY
$11 \times 1.45" = 15.95"$
$11 = (21-10)$

CEPTABLE SINCE $Q_{P2} < 0$.
UME $Q_{P2} = 0$ $STOR_2 = 0$

G STOR $= (15.95 + 0)/2 = 8$

$_3 = 27000\left(1 - \frac{8}{10}\right) = 5400$ CFS

400 CFS REQUIRES HEAD OF $12.6 - 10 = 2.6'$

$2.6' \times 1.45 = 3.77"$ OF STOR

$G = (8 + 3.77)/2 = 5.9"$

$Q_{P4} = \cancel{5400}\ 27000\left(1 - \frac{5.9}{10}\right) = 11070$ CFS

1070 CFS REQUIRES HEAD OF $15.5 - 10 = 5.5'$

$5.5' \times 1.45"/FT = 7.98"$

$5.9 + 8.0)/2 = 6.95 \approx 7.0"$

$Q_{P5} = 27000\left(1 - \frac{7}{10}\right) = 8100$ CFS

8100 CFS REQUIRES $14.2' - 10 = 4.2'$ OF HEAD

$4.2' \times 1.45"/FT = 6.09"$ OF RUNOFF

$G = (7 + 6.)/2 = 6.5"$

$Q_{P6} = 27000\left(1 - \frac{6.5}{10}\right) = 9450$ CFS

9450 CFS REQUIRES $14.8' - 10' = 4.8'$ OF HEAD

$4.8' \times 1.45"/FT = 6.96$ CLOSE ENOUGH

OUTFLOW $\approx$ 9450 CFS

BASED ON LOCAL RUNOFF OF 27,000 CFS

DAM FAILURE ANALYSIS:

ASSUME DAM FAILS WHEN WATER SURFACE IS AT SPILLWAY CREST WITH THE SLUICE GATES WIDE OPEN BUT 3' OF STOPLOGS IN PLACE. THE DISCHARGE JUST PRIOR TO FAILURE WOULD BE ≈ 3000 CFS

THE PEAK FLOW FROM FAILURE IS CALCULATED:

ASSUME 80' WIDE GAP OPENS

$$Q_{P1} = \frac{8}{27} W_b \sqrt{g}\, Y_0^{1.5} = \frac{8}{27}(80)\sqrt{32.2}\,(10)^{1.5}$$

$$Q_{P1} = 4250 \text{ CFS}$$

THUS THE PEAK FLOW DOWNSTREAM WOULD BE ≈ 7000 CFS.

THE FLOOD INSURANCE STUDIES FOR TILTON AND NORTHFIELD, N.H. USED A 500 YEAR FLOW OF 7320 AT LOCHMERE DAM AND 7670 CFS AT USGS GAUGE IN TILTON. THUS THE FLOOD DAMAGES FOR THE 500 YEAR EVENT ARE INDICATIVE OF THE DOWNSTREAM DAMAGE POTENTIAL FROM DAM FAILURE. GIVEN THE SUDDENNESS OF A DAM FAILURE WAVE SOME ATTENUATION IN SILVER LAKE COULD BE EXPECTED, BUT THE DAMAGES WOULD APPROACH THOSE OF THE 500 YR EVENT.

DAM FAILURE ANALYSIS: (CONTINUED)

THE FLOOD INSURANCE MAPS INDICATE THAT FOR A FLOW OF APPROX. 7000 CFS, THERE ARE THREE AREAS WHERE FLOODING OF STRUCTURES WOULD BE ANTICIPATED. THEY ARE ~~Along~~ ALONG THE SOUTHWEST SHORE OF SILVER LAKE, WHERE VARIOUS SUMMER COTTAGES ARE FOUND; JUST UPSTREAM OF THE ROUTE 140 BRIDGE WHERE A LIMITED NUMBER OF LOW LYING STRUCTURES ARE LOCATED, AND JUST UPSTREAM OF THE TILTON DAM WHERE OLD MILL AND INDUSTRIAL BLDGS ARE LOCATED ADJACENT TO THE RIVER.

APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS

PATENT PENDING Goldberg Zoino Dunnicliff + Assoc

INSERT FILM EMULSION SIDE DOWN

NEGATIVE PRESERVER DATE 4/13/78 ASSIGNMENT Lochmere Dam FILE NO. 2067

PHOTO PLASTIC PRODUCTS INC P.O. BOX 507 FAIRFAX VA